HYDRODYNAMIC MODELLING OF THE LOWER NERANG RIVER, SOUTH EAST QUEENSLAND, AUSTRALIA

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The Gold Coast city is located in a flood prone area of South East Queensland which has experienced more than 45 extreme flood events since 1924. The Nerang River catchment is the largest river system in the centre of the Gold Coast region, also a vulnerable area subject to flooding. Although, this river is significantly affected by tidal conditions and storms, limited study has been carried out on the water level changes in ungauged areas of the river. In the present study, the development of a 1D hydrodynamic modelling (MIKE HYDRO) along the tidal limit of the main Nerang River, and its calibration processes will be introduced. The model requires the Digital Elevation Model (DEM) and bathymetry data to generate cross sections along the river. Glenhurst Station’s discharge information and the Gold Coast Seaway observed tide are applied as upstream and downstream conditions, respectively. Water level records of Carrara Alert Station and Evandale Alert Station are applied for model calibration and verification, respectively. The model is calibrated based on Manning’s n coefficient and bathymetry data. The reliability of the results was investigated through calculating correlation coefficient, root mean square error, the Nash-Sutcliffe efficiency coefficient, mean absolute error, percent bias, and the RMSE standard deviation ratio. The present study outlined the steps and required input data for the Nerang River hydrodynamic modelling. The introduced model will help to understand the variations of water level, river discharge and flow velocity of the Nerang River. The results revealed that the calibrated Manning’s coefficient, ranging from 0.011-0.013, leads to a good agreement between observed data and simulated results. The validated model can be used for scenario studies and the investigation of the impacts of climate change for better city planning and more efficient management decision making.

Introduction

Accurate simulation of river flow is important in flood control operations since floods are one of the main causes of loss of life, particularly in flood prone areas. In order to reduce flood damage, hydrodynamic models are applicable to provide a physical basis for simulating a wide range of flow situations and sediment transport. The calculation of flow is an important task in river modelling as open channel flow is usually turbulent (Wu, et al., 2000). On the other hand, numerical simulation of flow is very challenging due to the complexity of turbulent flows, given the phenomenon of free water surface and movable channel bed (Wu, 2004).

Numerical models for flood simulation require solving the governing equations for river streamflows and flood plains. Patro et al. (2009) categorized the numerical models into three main groups: (i) one dimensional (1D) models, (ii) two dimensional (2D) models, (iii) 1D river flow models coupled with 2D floodplain flow (1D-2D) models. 1D hydrodynamic river models have been developed since the 1970s (Cunge et al., 1980), and there are well-known softwares for dynamic 1D river flow simulation such as HEC-RAS (Hydrologic Engineering Centre River Analysis System) by the US Army Corps of Hydraulic Engineers and Mike 11 by Danish Hydraulic Institute (DHI, 1997). Regarding the accurately described hydraulic behavior of natural rivers and streams, ease of use, and fewer costs, simple 1D hydrodynamic models are still common compared to higher dimensional models (Pappenberger et al., 2005). Additionally, Bates (2004) confirmed that in the case of using high resolution topographic data, 1D hydraulic models are able to simulate flood propagation and inundation extent of natural streams accurately. Furthermore, in terms of specific computational time, including real-time flood inundation...
forecasting, 1D hydraulic models are cheaper and more robust than 2D and 3D hydraulic models (Castellarin et al., 2009).

Studies of Southeast Queensland, Australia confirm that the Gold Coast is located in a flood prone area as during the last 120 years, more than 40 cyclones have affected the Gold Coast City (Mirfenderesk, 2009). The Nerang River, which supplies the drinking water of Gold Coast dwellers, is a tidal river encountering many flooding events, storm surges, and flood inundations. Therefore, modelling river flow and evaluating water level changes in the tidal limit of the river is essential to address storm damage through informing better city planning.

The present paper investigates the use of Mike Hydro (dynamic 1D) in the calibration and validation of the Nerang River tidal limit (from 23 kms upstream to the river mouth as shown in Figure 2). In this context, the selected boundary conditions for both upstream and downstream, required hydrological and topographic data, and detailed stages of model setup in Mike software are discussed. Next, the model is calibrated using the Manning’s roughness coefficient, n, and bathymetry data. The model performance indices and a correlation between observed and simulated data are analyzed to confirm the reliability of the model. Finally, the calibrated Manning’s roughness coefficient is used for model validation. The calculated performance indices of validation period confirm an evident agreement between observed data and simulated results.

**Methods**

**Study domain**

The Nerang River, located in the centre of the Gold Coast region, is approximately 250 km long and flows from McPherson Range and Springbrook Plateau, and covers an area of almost 493.3 Km² (Figure.1). This river consists of several waterways, and the total length of the waterway networks in the Nerang catchment is 928 kilometres where the Nerang River is the main waterway (Nerang River Catchment Study Guide, 2011). There are two dams along the Nerang River, including the Hinze Dam and the Little Nerang Dam, which supply drinking water. The catchment can be divided into three sub-catchments: the upper, middle, and lower reaches (Mitchell and Oldridge, 2006). Additionally, 13 sub catchments are located in the upper area of the Nerang River catchment, containing fresh water only, and the lower sub catchment and estuarine areas below the Hinze Dam spillway have poorer water quality (Nerang River Catchment Study Guide, 2011).
Data collection and description

In the present paper, the lower Nerang River dominated by the tide motion from the tidal limit to the river mouth, has been selected for modelling. The Queensland Government Water Monitoring Information Portal (WMIP) provides time-varying streamflow and water level data from Glenhurst monitoring station (153°18’36.1” E, 27°59’59.9” S), as marked in Figure 2. This stream monitoring station is located at the tidal limit of the Nerang River (22.7 kms away from the river mouth). In order to calibrate and validate the model, unregulated water level records of Carrara Alert Station (14.7 km away from mouth river) and hourly water level records of Evandale Alert station (3.8 km away from mouth river) have been obtained from the Department of Environment and Resources Management (DERM) and the Bureau of Meteorology (BOM) of the Australian Government for Queensland.

Hydrodynamic river models need river cross section data, in which the topographic elevations and water depth data are obtained from the Digital Elevation Model (DEM) and bathymetry data, respectively. The Australian Government Geoscience Australia provides the mentioned data. The DEM of the study domain with a grid size of five metres is utilized to depict the topography level along the Nerang River (Figure 3). The DEM is located under the GDA_1994_MGA_Zone_56 coordinate system with reference to the datum D_GDA_1994. Additionally, to improve the accuracy of the cross sections along the river, the bathymetry data with a grid size of 30 m are applied.
In order to consider the impacts of tides on the river flow, the hourly observed tide at the Gold Coast Seaway has been obtained from the BOM. The datum of the provided data is LAT (lowest astronomical tide), while the datum of other observed data is AHD (Australian Height Datum). Therefore, based on “standard port datum levels”, AHD is above LAT with the value of 0.76 metres. To convert tide records data to AHD datum, the following relation is used:

\[ AHD \text{ Records} = LAT \text{ Records} - 0.76 \]  

(1)

Figure 2. Study Domain (Google Earth, 2017)

Figure 3. The DEM (in meters) of the study domain
Model setup

Theoretical background

We performed the hydrodynamic modelling, which solves the Saint-Venant governing equation, as follows:

\[
\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \tag{2}
\]

\[
\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \alpha \frac{Q^2}{A} \right) + gA \frac{\partial h}{\partial x} + \frac{n^2 g |Q|}{AR^{1/3}} = 0 \tag{3}
\]

where \( Q \) = discharge (m\(^3\)/s); \( A \) = cross section flow area (m\(^2\)); \( q \) = lateral inflow (m\(^2\)/s); \( h \) = water level above a reference datum (m); \( x \) = downstream direction (m); \( t \) = time (s); \( n \) = Manning resistance coefficient (s/m\(^{1/3}\)); \( R \) = hydraulic or resistance radius (m); \( g \) = gravity acceleration (m/s\(^2\)); and \( \alpha \) = momentum distribution coefficient (DHI, 2016).

1D Mike Hydro setup

Modelling and unsteady simulation of the lower Nerang River were undertaken with the aid of Mike Hydro, which is an implicit finite difference mathematical model for simulating unsteady flow in rivers and floodplains in shallow water types (DHI, 2016).

Regarding model setup, the river network should initially be digitized using a topographic base map and DEM data. Next, the cross sections data are extracted along the river in order to analyse the results at different points of the river network. During the next stage, the initial conditions of the river should be set up by defining simulation period (from 1/1/2012 14:00 to 1/12/2012 0:00), time steps (2 minutes), and initial water level at the beginning of the simulation period (Carrara water level=0.53m). The most important step of 1D hydrodynamic model setup is defining accurate boundary conditions, including both upstream and downstream conditions to eliminate errors of inflow and outflow. In the present case study, hourly water level records of Glenhurst monitoring site in 2012 (Figure 4) are used as the time varying upstream open boundary condition, while hourly observed Gold Coast Seaway tides over the same year are defined as the time varying downstream open boundary condition. The observed tide values range from -0.933 m to 1.32 m over the year 2012, and the mean water level is 0.116m during the same period.

![Figure 4. Hourly water level at Glenhurst Station](2018 Floodplain Management Australia National Conference-page 5)
Model calibration and validation

In hydrodynamic modelling, calibration is a process of varying the friction coefficient values to obtain the optimum levels of agreement between observations and model predictions. In the present paper, the model is calibrated using the hourly observed water levels from Carrara Alert site (1/1/2012-8/4/2012) (Fig 5). Moreover, Manning’s coefficient friction and the hourly observed water level records of Evandale Alert station (over five days) (Fig 6) are applied to validate the model. The data available for model calibration and validation are the water level time series of two gauging sites, as stated earlier. Subsequently, the performance indices, including correlation coefficient ($R^2$), root mean square error (RMSE), Nash-Sutcliffe efficiency coefficient (NSE), mean absolute error (MAE), percent bias (PB), and RMSE standard deviation ratio (RSR) are calculated to test the reliability of the model.

![Figure 5. Hourly water level at Carrara Alert station](image1)

![Figure 6. Hourly water level at Evandale Alert station](image2)

Performance indices

In the present research, six performance indices are estimated to analyse the accuracy of the results.

1. $R^2$ is defined as shown below:
\[ R^2 = \frac{1}{N-1} \sum_{i=1}^{n} (S_i - \mu_s)(O_i - \mu_o) \sigma_s \sigma_o \]  \hspace{1cm} (4)

where, \( N \) = total number of observations; \( \mu_s \) and \( \mu_o \) = average of simulated and observed water level, respectively; \( O_i \) = observed water level at the \( i \)th hour; and \( S_i \) = simulated water level at the \( i \)th hour; and \( \sigma_s \) and \( \sigma_o \) = standard deviation of the simulated and observed water levels, respectively.

(2) \( \text{RMSE} \) is defined as follows:

\[ \text{RMSE} = \sqrt{\frac{1}{N} \sum_{i=1}^{N} (O_i - S_i)^2} \]  \hspace{1cm} (5)

(3) \( \text{NSE} \) is calculated as below:

\[ \text{NSE} = 1 - \frac{\sum_{i=1}^{N} (O_i - S_i)^2}{\sum_{i=1}^{N} (O_i - \mu_o)^2} \]  \hspace{1cm} (6)

(4) \( \text{MAE} \) is defined as below:

\[ \text{MAE} = \frac{\sum_{i=1}^{N} |O_i - S_i|}{N} \]  \hspace{1cm} (7)

(5) \( \text{PB} \) is defined as below:

\[ \text{Pbias} = \frac{\sum_{i=1}^{N} (O_i - S_i)}{\sum_{i=1}^{N} O_i} \times 100 \]  \hspace{1cm} (8)

(6) \( \text{RSR} \) is estimated as below:

\[ \text{RSR} = \frac{\text{RMSE}}{\sigma_o} = \frac{\sqrt{\sum_{i=1}^{N} (O_i - S_i)^2}}{\sqrt{\sum_{i=1}^{N} (O_i - \mu_o)^2}} \]  \hspace{1cm} (9)

Results and discussion

Elevation refinement of DEM extracted cross sections

Regarding the use of water level records as input data, the cross section elevations at different locations of the river play a significant role in hydrodynamic modeling. 101 cross sections were created along the Nerang River from chainage 0 (upstream boundary) to
chainage 22.50×10³ m (downstream boundary). These values were compared with (i) a sample cross section from Glenhurst station (provided by WMIP), (ii) the Nerang River navigation map (Department of Transport and Main Roads, Queensland Government), and (iii) bathymetry data. As a result of such comparisons, it is observed that the river depth values of the extracted cross sections were lower than the water depth values of the river, even though river depth should be higher than water depth. This mismatch happened due to the fact that the riverbed of the DEM extracted cross sections were identified as indicating the real water surface rather than riverbed values (Figure 7). That is why bathymetry data is required to modify the DEM extracted cross sections. Therefore, all the DEM derived cross section values were adjusted using the bathymetry data. To amend the extracted cross sections values, at the same location, the difference between the DEM-derived cross section values and water depth values (bathymetry data) should be added to the DEM-derived cross section elevation values to obtain accurate cross sections. As a result, the initial obtained riverbed should shift downward to create the accurate cross sections.

![Figure 7. Illustration of the cross section modification](image)

**Calibration of Mike hydrodynamic 1D model in Mike Hydro**

After setting up the model, the model calibration was conducted by comparing observed water records and simulated water levels, such as those available from Carrara Alert station (chainage 7.52×10³ m) in 2012. The calibration parameter is Manning’s roughness coefficient (n) of the riverbed. Initially, the model was simulated using the default value of the Manning’s roughness coefficient (n=0.0333) in Mike Hydro. During the calibration process, the Manning’s roughness coefficient was modified to derive the best agreement between the observed and simulated water levels at Carrara Alert site. The calibrated Manning’s n values for different cross sections are presented in Table 1.

<table>
<thead>
<tr>
<th>Table 1. Manning’s roughness coefficient for the Nerang River chainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>River Chainage (m×10³)</td>
</tr>
<tr>
<td>------------------------</td>
</tr>
<tr>
<td>0-7.52</td>
</tr>
<tr>
<td>7.62-22.50</td>
</tr>
</tbody>
</table>

The simulated water levels at Carrara Alert site is compared to observed records in Figure 8 for three periods: (a) 2/1/2012-6/1/2012, (b) 23/1/2012-27/1/2012, and (c) 24/2/2012-28/2/2012. As can be seen, Carrara Alert site is highly dominated by tidal influence, and the defined downstream boundary condition has a significant impact on the computed water levels of at Carrara Alert site. According to Figure 8, the simulated water levels are higher than those observed in most periods. This is due to the existence of canals and small branches reaching the Nerang River at different locations; these
branches and canals are not considered in the present research. Currently, limited available data at reaching points and canals have led to the existence mangrove canals being neglected in the present study, which is the main reason for water levels overestimation. However, a clear agreement can be seen during the low-flow periods.

Figure 8. Comparison of observed and simulated water levels at Carrara Alert site period (a), (b), and (c)
Table 2 shows the estimated performance indices of the simulated results at Carrara site, which confirm the reliability of the model. Pan et al., (2013) and Cho et al., (2013) have defined performance ratings for NSE, PB, and RSR. Based on their classification, $0.75<\text{NSE} \leq 1$, $PB \leq 10$, and $0 \leq RSR \leq 0.5$ are rated “very good”, $0.65<\text{NSE} \leq 0.75$, $10 \leq PB \leq 15$, and $0.5<\text{RSR} \leq 0.6$ are rated “good”, $0.5<\text{NSE} \leq 0.65$, $15 \leq PB \leq 25$, and $0.6<\text{RSR}<0.7$ are rated “satisfactory”, and $\text{NSE} \leq 0.5$, $PB \geq 25$, and $RSR \geq 0.7$ are rated “unsatisfactory”.

Table 2 shows that there is a good correlation between simulated and observed water levels as, typically, $R^2$ values greater than 0.5 are considered acceptable (Santhi et al., 2001 and Van Liew et al., 2003). NSE is rated as very good, satisfactory, and good for periods a, b, and c respectively. Period b refers to a flooding event, which is why it is classified in a satisfactory range rather than being in a good range. MAE indicates the average magnitudes of the errors, which are 0.1, 0.23, and 0.14 m for periods a, b, and c respectively. RMSE, PB, and RSR performance criteria are classified in the satisfactory range as well. In general, there is a close agreement between observed and simulated water levels, while the mentioned errors are due to neglecting canals and branches.

<table>
<thead>
<tr>
<th>Performance indices</th>
<th>Period a</th>
<th>Period b</th>
<th>Period c</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R^2$</td>
<td>0.90</td>
<td>0.82</td>
<td>0.85</td>
</tr>
<tr>
<td>RMSE (m)</td>
<td>0.12</td>
<td>0.33</td>
<td>0.20</td>
</tr>
<tr>
<td>NSE</td>
<td>0.80</td>
<td>0.58</td>
<td>0.67</td>
</tr>
<tr>
<td>MAE (m)</td>
<td>0.10</td>
<td>0.23</td>
<td>0.14</td>
</tr>
<tr>
<td>PB</td>
<td>-24.38</td>
<td>-12.05</td>
<td>17.78</td>
</tr>
<tr>
<td>RSR</td>
<td>0.45</td>
<td>0.65</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Validation of Mike hydrodynamic 1D model in Mike Hydro

After the model calibration, the Mike Hydro model was validated using the hourly water level records of Evandale Alert site (chainage 18552.3m) over five days (31/5/2012 1:00 AM-3/6/2012 6:00 AM). This period was selected due to limited available data. Figure 9 indicates the compared water level values of the observation and simulation at Evandale Alert station during the validation period. It is apparent that both water level values fit very well.

Figure 9. Comparison of hourly observed and simulated water levels at Evandale Alert site over five days
The performance indices of the validation period are presented in Table 3. Based on the stated performance rating, NSE, PB, and RSR are rated in the “very good” range. This shows that the calibrated model performs reasonably during the validation period as well.

Table 3. Performance indices for Evandale Alert site during calibration for the year 2012

<table>
<thead>
<tr>
<th>Performance indices</th>
<th>Estimated Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R^2$</td>
<td>0.98</td>
</tr>
<tr>
<td>RMSE (m)</td>
<td>0.1</td>
</tr>
<tr>
<td>NSE</td>
<td>0.93</td>
</tr>
<tr>
<td>MAE (m)</td>
<td>0.074</td>
</tr>
<tr>
<td>$PB$</td>
<td>-9.33</td>
</tr>
<tr>
<td>$RSR$</td>
<td>0.26</td>
</tr>
</tbody>
</table>

Conclusion

A review of studies revealed that hydrodynamic modeling of rivers, particularly in flood prone regions, such as the Gold Coast City is complex due to turbulent and unsteady flow. Considering many reported flooding events on the Gold Coast and in the Nerang River, hydrodynamic model of the stated river flow can potentially provide helpful information for city planning. Therefore, in the present paper an effort has been made to establish a 1D hydrodynamic model in Mike Hydro. During the setup process, DEM-derived cross sections values are contradicted with by sample Glenhurst cross section and bathymetry data. To modify the DEM-derived cross sections, the difference between DEM-derived cross section elevation values and water depth was estimated and then, this calculated difference was added to the DEM-derived cross section elevation values to obtain accurate cross sections. The model was set for the year 2012 simulation and for defining two boundary conditions upstream (Glenhurst site) and downstream (Carrara Alert station). Manning’s roughness coefficient $n$ of the riverbed was selected as the calibration parameter, and the model was calibrated by comparing the water level records at Carrara Alert site and by calculating performance indices. The optimal Manning’s roughness coefficient ranging 0.011-0.013, which leads to a reasonable agreement between observed and computed water level values. In the flooding period, the computed water levels were overestimated compared to the recorded water levels at Carrara Alert site. The reason for this overestimation is neglecting the canals and branches of the Nerang River due to limited available data. However, there is substantial agreement during the low-flow periods. During the validation process, water level records of Evandale Alert site were applied, which show high agreement between the hourly measured and simulated results over five days.

According to the presented results, in spite of the use of limited data, the calibrated model performs quite satisfactorily in simulating Nerang River flow. The model is also concluded to perform well in simulating low flow condition as well as tidal condition since the lower Nerang River is located at the tidal limit and is highly influenced by tides. However, the simulated peak flows appear to return to their normal condition slowly, which is due to the necessity of ignoring the mangrove canals and small branches. The authors aim to investigate the impacts of mangrove canals on river conditions in the validated model to improve the results.

The introduced model can be used to understand the variations of water level, river discharge and flow velocity of the Nerang River. It can also help to evaluate flood events and resulted water level changes along the river. Therefore, water level conditions of the different points along the Nerang River, which have no field data, can be estimated using the proposed hydrodynamic model. Additionally, present model can be applied to 2018 Floodplain Management Australia National Conference-page 11.
analyse the impact of sea level rise due to climate change, which will be investigated in further studies.

Acknowledgement

The authors would like to acknowledge the support of the Water Monitoring Information Portal, Queensland Government, Australia in their provision of the water level data, and of the Bureau of Meteorology, Australia for providing Gold Coast Seaway tide data. Additionally, the Department of Environment and Resources Management provided water level records at Evandale Alert site, and Geoscience Australia provided the DEM data.

References

